SCI PUBLICATION 056

The Fire Resistance of Composite Floors with Steel Decking (2nd Edition)

Der Feuerwiderstand von verbunddecken mit Stahltrapezprofilen (2. Auflage)

La résistance à l'incendie des planchers composites avec tôle profilée en acier (2e édition)

Resistencia al fuego de forjados compuestos con chapa de acero (2ª Edición)

G M NEWMAN BSc(Eng), CEng, MIStructE, MSFSE

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The Steel Construction Institute
Silwood Park, Ascot
Berkshire SL5 7QN
Telephone: 0344 23345
Fax: 0344 22944

Offices also at:

Unit 820, Birchwood Boulevard
Birchwood, Warrington
Cheshire WA3 7QZ

B-3040 Huldenberg
52 De Limburg Stirumlaan
Belgium

Foreword

This publication provides information on the two methods commonly used for verifying the fire resistance of composite floors. It builds on the earlier work of the Constructional Steel Research and Development Organisation and incorporates developments that have stemmed from recent research. It has been prepared by Mr G M Newman of the Steel Construction Institute.

The following commented on the text and the design examples in the first edition of this publication:

Dr. G.M.E. Cooke	Fire Research Station, BRE
Dr. R.M. Lawson	The Steel Construction Institute
Mr. F.P.D. Ward	Richard Lees Ltd
Mr. G. Hogan	British Steel
Mr. E. Hindhaugh	British Steel
Mr. P.J. Wickens	Mott, Hay and Anderson, Structural and Industrial Consultants

The methods described are referred to by BS 5950: Part 8: 1990 *Code of Practice for Fire Resistant Design*. Mr. Newman, Dr. Lawson and Dr. Cooke were members of the drafting committee of that Standard.

The Second Edition includes new research information based on tests carried out in 1990. This has resulted in a number of recommendations on the fire protection of beams supporting composite floors.

The continuing support of British Steel in the preparation of this publication is acknowledged.

Contents

SUMMARY

The Fire Resistance of Composite Floors with Steel Decking (2nd Edition)

This publication describes two methods for verifying the fire resistance of composite floors. In the fire engineering method a calculation procedure is described to assess the structural performance in fire using any arrangement of reinforcement. In the simplified method rules are given which allow the use of standard reinforcing meshes with little or no calculation. Both approaches are referred to by BS 5950: Part 8: 1990 *Code of Practice for Fire Resistant Design.*

New research carried out by the SCI in 1990 resulted in a number of recommendations being made for the fire protection of beams supporting composite floors. A summary of these recommendations has been included in the 2nd edition. An important conclusion was that the voids formed between the underside of the steel deck and the top flange of the beam may often be left unfilled.

Der Feuerwiderstand von verbunddecken mit Stahltrapezprofilen (2. Auflage)

Zusammenfassung

Diese Veröffentlichung beschreibt zwei Methoden zur Überprüfung des Feuerwiderstands von Verbunddecken. Die 'Fire-Engineering-Methode' beschreibt ein Berechnungsverfahren zur Beurteilung des Tragwerkverhaltens im Brandfall bei beliebiger Anordnung von Bewehrung. Die vereinfachte Methode erlaubt den Einsatz von gewöhnlicher Mattenbewehrung mit geringem, oder ohne, Rechenaufwand. Beide Verfahren beziehen sich auf BS 5950, Teil 8, 1990: Code of Practice for Fire Resistant Design.

Neue Forschungen, die 1990 vom SCI durchgeführt wurden, führten zu einer Reihe von Empfehlungen hinsichtlich des brandschutzes von Verbunddeckenträgern. Eine Zusammenfassung dieser Empfehlungen ist in der zweiten Auflage enthalten. Eine wichtige Schlußfolgerung war, daß die Hohlräume zwischen Trapezprofil und Trägeroberflansch oft offen bleiben können.

La résistance à l'incendie des planchers composites avec tôle profilée en acier (2e édition)

Résumé

La publication décrit deux méthodes de vérification à l'incendie des planchers composites. Dans la méthode d'ingénieur, une procédure de calcul est exposée qui permet d'atteindre, sous incendie, les performances structurales pour n'importe quel type de renforcement. Dans la méthode simplifiée, des régles sont proposées qui permettent, pratiquement sans calcul, d'utiliser des renforts standards. Les deux approches se réfèrent à la BS 5950: Partie 8: 1990 - Code de pratique pour le dimensionnement sous incendie.

Une nouvelle recherche menée, en 1990, par le SCI a conduit à diverses recommandations concernant la protection à l'incendie de poutres supportant des planchers composites. Un résumé de ces recommandations est inclu dans cette 2e édition. Cette recherche a conduit à la conclusion, importante, que les vides existants entre le côté inférieur de la tôle profilée et la semelle supérieure des poutres peut souvent être laissé sans remplissage.

Resistencia al fuego de forjados compuestos con chapa de acero (2ª Edición)

Resumen

Esta publicación describe dos métodos para comprobar la resistencia al fuego de forjados compuestos. En el método ingenieril se describe un procedimiento para calcular el funcionamiento de la estructura ante el fuego usando una distribución de armado arbitraria. En el método simplificado se aconsejan disposiciones de mallas de armado tipo prácticamente sin ningún cálculo. Ambas alternativas se refieren a la Norma BS 5950: Parte 8: 1990: Norma para el Diseño con Resistencia al Fuego.

Debido a nuevas investigaciones desarrolladas en el Steel Construction Institute, en 1990 se propusieron nuevas recomendaciones para la protección ante el fuego de vigas en forjados compuestos. En la segunda edición se ha incluido un resumen de estas recomendaciones una de cuyas conclusiones más importantes fue que los huecos formados entre la chapa de acero y el ala superior de la viga pueden, a menudo, dejarse sin rellenar.

Notation

D	Depth of deck profile
D_s	Overall slab depth
f_{cu}	Characteristic cube strength of concrete
f_y	Reinforcement yield strength
K_r	Material strength reduction factor
L	Span of floor
M_S	Moment capacity of section resisting sagging
M_H	Moment capacity of section resisting hogging
M_o	Free bending moment
w_s	Self weight of composite floor per unit area
w_d	Total dead load per unit area
w_i	Total imposed load per unit area
p_r	Design strength of reinforcement
p_c	Design strength of concrete
γ_{mc}	Concrete material strength factor
γ_{mr}	Reinforcement material strength factor
γ_{fd}	Load factor for dead loads
γ_{fi}	Load factor for imposed loads
MFD	Moment depth factor
t	Steel deck thickness

1. INTRODUCTION

Since the publication of the original Steel Construction Institute's Recommendations in 1983[1] much research has been carried out in the UK into the behaviour of composite steel deck floors in fire. This research has shown that the original recommendations were generally conservative and that it may not always be necessary to carry out a fire engineering calculation to verify the fire resistance in many common situations.

This publication describes two methods of verifying the fire resistance of composite steel deck floors. The first of these is a calculation method based on the theoretical behaviour of composite floors in fire and is generally the same as the method given in the original recommendations. The second method (the simplified method) has evolved from recent research and can be used for a given range of spans and loadings to provide up to 2 hours fire resistance. It depends on the use of a single layer of standard reinforcing mesh.

The publication also contains guidance on the fire resistance of composite beams. Since the first edition a research programme has been carried out and an SCI Technical Report[14] has been published. The recommendations of that report are summarised in Section 6.

2. COMPOSITE STEEL DECK FLOORS

Modern steel framed multi-storey buildings commonly use composite steel deck floors. These floors consist of a profiled steel deck with a concrete topping. Included within the concrete is some light reinforcement (see Figure 1). Indentations in the deck enable the deck and concrete to act together as a composite slab. The reinforcement is included to control cracking, to resist longitudinal shear and, in the case of fire, to act as tensile reinforcement. It is normal to extend the composite action to the supporting beams. Shear studs are welded through the deck onto the top flange of the beam to develop composite action between the beam and concrete slab. The resulting, two-way-acting, composite floor is structurally efficient and economic to construct.

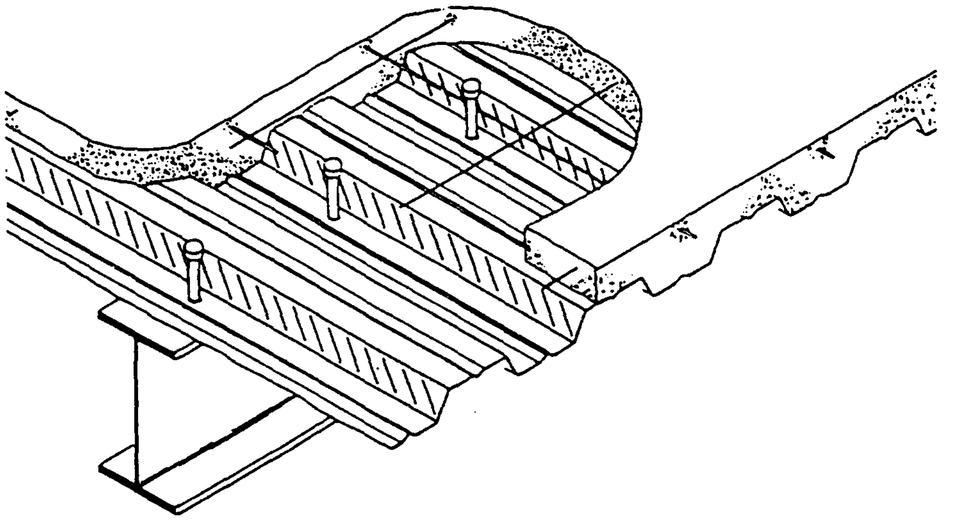


Figure 1 *The principal components of a composite floor*

The design of the composite slab is governed by BS 5950: Part 4[2]. The design of the composite beams is governed by BS 5950: Part 3[3]. The Steel Construction Institute have prepared design recommendations for composite beams[4].

Composite steel deck floors are almost invariably used without any fire protection to the exposed steel soffit although the supporting beams are fire protected. It is this exposure of the deck, which normally acts as tensile reinforcement, that leads to special consideration of the fire performance of these systems. BS 5950: Part 8: 1990[5] gives guidance on the fire resistance of floors and refers to the methods described in this publication.

Fire resistance is achieved by including reinforcement within the floor slab. At the high temperatures reached in fires the contribution of the steel deck to the overall strength is small and is normally neglected. The resulting approach follows the methodology used in ordinary reinforced concrete design in that concrete is used as "insulation" to keep the reinforcement at a temperature at which it can support the applied load. However, in most circumstances, because the cover to the reinforcement is greater than that which would be used in ordinary reinforced concrete design, the temperature reached by the reinforcement will be correspondingly lower. No spalling of the concrete occurs.

The methods described in this publication are apparently conservative in comparison with test performances associated with construction methods in the USA[6] and Canada. This is illustrated by the fact that in the USA it is normal to use the equivalent of D49 wrapping fabric (2.5 mm diameter wires at 100 mm centres), whereas the methods described here would normally result in at least three times that area of reinforcement. However, in those countries the method of testing is very different to UK and European practice.

Fire resistance tests in North America are "restrained" tests in that the specimen is constrained within a frame which is able to resist thermal expansion. This may simulate behaviour near the middle of a floor but may not be representative of edge conditions. However, although in North America less reinforcement is used for a given period of fire resistance than is normally used in the UK, comparable buildings are required to have higher fire resistance in North America than in the UK.

3. FIRE TESTS ON COMPOSITE STEEL DECK FLOORS

Since the publication of the earlier Steel Construction Institute Recommendations[1] many fire resistance tests have been carried out in the UK. These tests were designed firstly to gain the acceptance of these unprotected composite floors by the regulating authorities, and secondly, to verify the rules for designing the reinforcement.

Two main series of tests have been carried out. British Steel, supported by the Fire Research Station, carried out three tests incorporating normal and lightweight concrete with open trapezoidal and closed dovetail steel decks. The tests were designed to model the corner of a building (see Figure 2). The test construction measured 7.2 m by 4.1 m and consisted of two 3 m spans with a cantilever to develop further continuity. In an attempt to model the behaviour of the full 8 m span beams, a sliding joint was used on the edge beam. This allowed the edge beam to pull in as the slab deflected. Cranked reinforcement was used (see Figure 3) and each test was designed to have 60 minutes fire resistance using the methods given in Reference 1.

The Construction Industry Research and Information Association (CIRIA) carried out a series of six tests to investigate the use of standard reinforcement mesh for up to 3.6 m spans and total imposed loads of up to 6.7 kN/m². One of these tests was similar to the BSC/FRS tests while the remaining tests had a main span of 3 or 3.6 m and a short span, loaded by a hydraulic jack to simulate continuity.

More recently a number of decking manufacturers have carried out tests. A summary of the main features of the fire tests is given in Table 1 and a detailed analysis of much of the test data is given in Reference 7.

Testing of all the slabs was carried out after storing for 5 to 6 months in dry conditions. This was to ensure that the moisture content of the concrete was representative of its in-service condition. Failure to do this would have resulted in optimistic fire resistances because large amounts of heat are required to dry out the concrete.

The final moisture content of the lightweight concrete was 4.0 to 6.9% by weight and that of the normal weight concrete was 3.5 to 4.5%. These moisture contents are not considered excessive. The concrete was in all cases of nominal grade 30. The supporting steel beams were fire protected to give at least 2 hours fire resistance.

The series of tests demonstrated that the original recommendations were generally conservative especially in respect of the requirements of overall slab thickness. They also demonstrated that in certain circumstances a fire engineering approach is unnecessary.

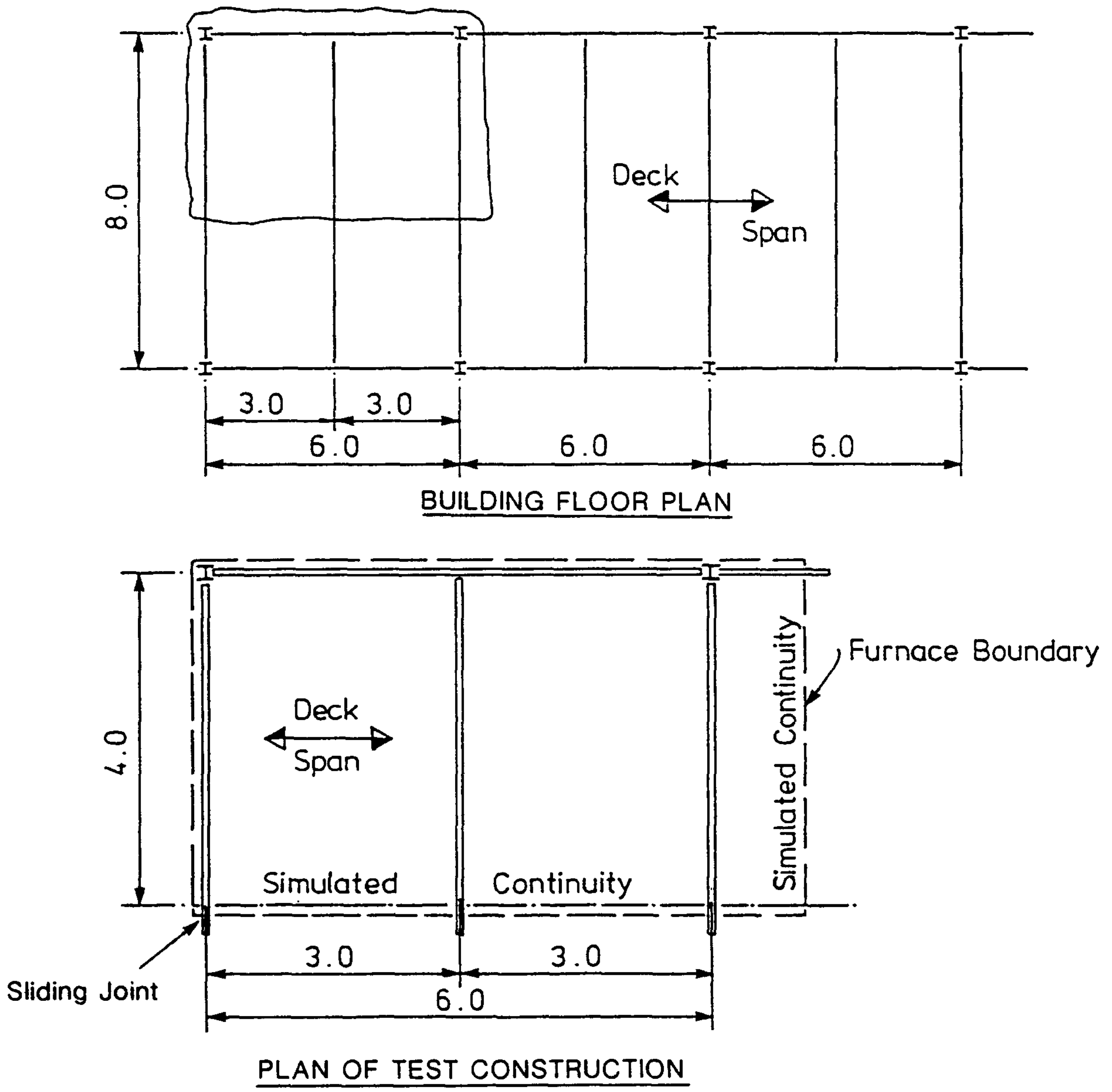


Figure 2 *BSC/FRS fire test simulating the corner of a building*

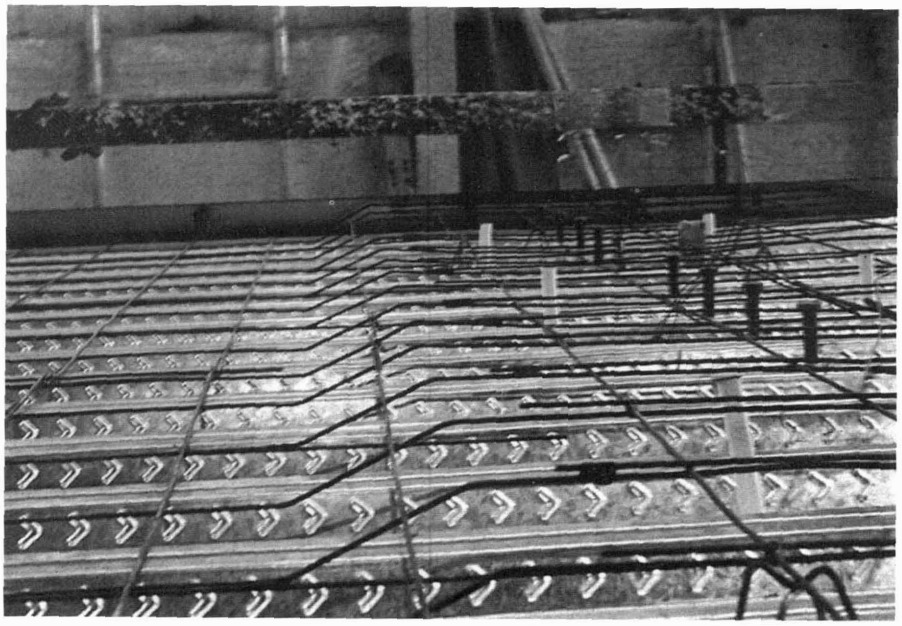


Figure 3 *Fire test specimen employing cranked mesh under construction*

TABLE 1 *Summary of UK fire tests on composite slabs*

PROFILE	CONCRETE TYPE	SLAB DEPTH (mm)	SPAN (m)	IMPOSED LOAD (kN/m²)	REINFORCEMENT	SURFACE TEMP. (°C)		TEST PERIOD (min)	TEST REF.
						AFTER 1 h	AFTER 1½ h		
Robertson QL59	LWC	130	3.0s	6.7	A142 mesh	73	-	60	CIRIA 1
Robertson QL59	LWC	130	3.0c*	6.7	A142 mesh	70	100	105	CIRIA 2
Robertson QL59	LWC	130	3.0c	6.7	A142 mesh	95	110	90	CIRIA 3
Holorib (UK)	LWC	120	3.0c*	6.7	A142 mesh	60	100	90	CIRIA 4
PMF CF46	LWC	110	3.0c	5.25	Y5 @ 225 as mesh	110	135	101	FRS-BS1
Holorib (UK)	LWC	100	3.0c	5.75	Y5 @ 150 as mesh	90	120	87†	FRS-BS2
PMF CF46	NWC	135	3.0c	6.75	Y5 @ 225 as mesh	85	95	120	FRS-BS3
Robertson QL59	NWC	140	3.6c*	6.7	A193 mesh	66	98	90	CIRIA 5
Metecno A55	NWC	140	3.6c*	6.7	A193 mesh	65	95	90	CIRIA 6
Holorib (UK)	LWC	150	3.0c*	10.0	A193 mesh	45	61	120	R.LEES 1
Ribdeck 60	LWC	140	3.0c*	5.6	A193 mesh	64	93	136	R.LEES 2
Ribdeck 60	LWC	140	3.0c*	8.5	A252 mesh	56	77	149	R.LEES 3
Alphalok	LWC	130	3.6c*	6.7	A252 mesh	92	110	128	ALPHA 1
SMD R51	NWC	140	3.0c*	6.7	A193 mesh	96	102	135	SMD 1
Quikspan Q51	NWC	140	3.0c*	5.0	A142 mesh	52	78	126	QUIK 1
Quikspan Q60	NWC	150	3.0c*	5.0	A142 mesh	79	97	122	QUIK 2
Multideck 60	NWC	150	3.6c*	6.7	A252 mesh	74	89	135	WARD 1
Multideck 80	NWC	150	4.0c*	6.7	A252 mesh	69	87	92	WARD 2

The tests are in chronological sequence from July 1983 until July 1991
Surface temperatures are the average values on the unexposed surface

† *failed prematurely because of the loss of protection to beams*
* *tests on long span/short span configuration*

s = simply supported c = continuous slab test

4. STRUCTURAL BEHAVIOUR IN FIRE

A composite steel deck floor is designed in bending as either a series of simply supported spans or as a continuous slab. In fire the floor may be considered to be simply supported or continuous regardless of the basis of the initial design. Strength in fire is ensured by the inclusion of sufficient reinforcement. This can be the reinforcement present in ordinary (room temperature) design and it is not necessarily additional reinforcement included solely for the fire condition.

During a fire the steel deck heats up rapidly, expands and may possibly separate from the concrete. However, in recent tests debonding of the deck was not significant. It is normal, although conservative, to assume that it contributes no strength in fire. The deck does, however, play an important part in improving the integrity and insulation aspects of the fire resistance: it acts as a diaphragm preventing the passage of flame and hot gases, as a shield reducing the flow of heat into the concrete, and it controls spalling.

With the strength of the deck discounted, the reinforcement becomes effective and the floor acts as a reinforced concrete slab with the loads being resisted by the bending action of the slab. Eventually the reinforcement yields and the slab fails. Catenary action may develop away from the edges of the floor with the reinforcement, assisted to a small extent by the steel deck, acting in direct tension rather than bending. An important conclusion from the recent tests is that the deformation of supporting edge beams is minimal and that catenary action is very small. The apparent shortening of span due to downwards central deflection is approximately equal to the increase in span due to thermal expansion.

The role of the concrete is very important in that it insulates the reinforcement and controls the transmission of heat through the floor. In both these respects lightweight aggregate concrete has a better performance than normal weight concrete. Lightweight concrete also loses strength less rapidly than normal weight concrete in a fire.

5. DESIGN FOR FIRE RESISTANCE

5.1 BS 476 Requirements

Fire resistance is expressed in terms of compliance with BS 476: Part 20 and Part 21[8]. It is a measure of the time before an element of construction exceeds the limits for load carrying capacity, insulation and integrity. These limits are fully defined in the Standard. They may be summarised as follows:

a) *Load carrying capacity*

The ability to support the test load whilst deflection is limited to span/20 and the rate of deflection does not exceed:

span^2/9000d mm per minute

where d is the distance from the top of the structural section to the bottom of the design tension zone. All dimensions in mm.

The rate of deflection criterion is not applied until the maximum deflection exceeds span/30.

b) *Insulation*
The ability to limit the conduction of heat to the upper surface. The average rise in temperature of the upper surface should not exceed 140°C and the maximum rise in temperature should not exceed 180°C.

c) *Integrity*
The ability to resist the passage of flame and hot gases.

Compliance with (c), integrity, is ensured with composite steel deck floors by the combined action of the diaphragm formed by the steel sheet and the reinforced concrete. Compliance with (b), insulation, is ensured by the provision of an adequate thickness of concrete. This may be obtained from Tables 2 and 3 for a fire engineering design or Tables 6 and 7 if the simplified method is used. Tables 2 and 3 should be read in conjunction with Figures 4 and 5 respectively.

Compliance with (a), load carrying capacity, is discussed below.

Table 2 *Minimum insulation thickness of concrete for trapezoidal decks*

Fire resistance period (hours)	Minimum insulation thickness of concrete (mm)	
	Normal weight concrete	Lightweight concrete
½	60	50
1	70	60
1½	80	70
2	95	80
3	115	100
4	130	115

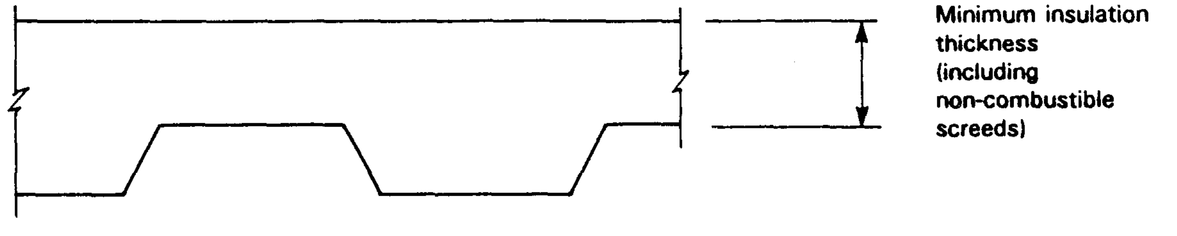


Figure 4 *Measurement of minimum insulation thickness of concrete for trapezoidal decks*

Table 3 *Minimum insulation thickness of concrete for re-entrant profile decks (equals overall slab depth)*

Fire resistance period (hours)	Minimum insulation thickness of concrete (mm)	
	Normal weight concrete	Lightweight concrete
½	90	90
1	90	90
1½	110	105
2	125	115
3	150	135
4	170	150

BS 5950: Part 4 specifies a minimum concrete cover to the deck of 50 mm.

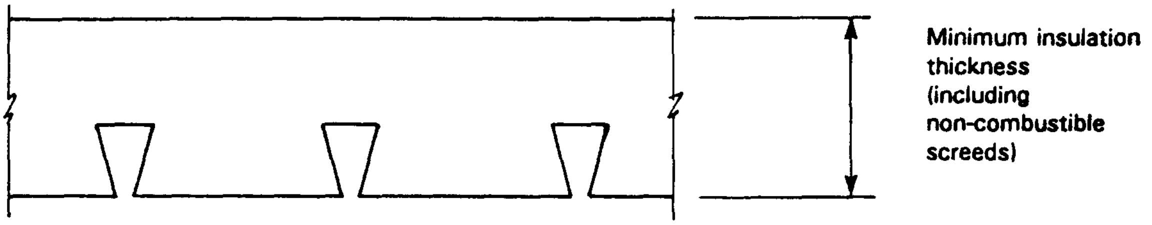


Figure 5 *Measurement of minimum insulation thickness of concrete for re-entrant profile decks*

5.1.1 Load Carrying Capacity

The load carrying capacity of the floor at the temperatures likely to be reached at the end of a fire test may be demonstrated using the fire engineering method or may be considered to be adequate provided the conditions of the simplified method are followed.

Tests have shown that floors designed using these methods perform well in fire tests and achieve fire resistance times greater than predicted. In the tests the span/20 deflection limit governs, and the rate of increase of deflection is rarely critical. Shear failure has never been observed and for design purposes may be neglected. It is considered that the rate of loss of bending strength in fire will be greater than the rate of loss of shear strength.

It is, therefore, considered sufficient to demonstrate that the floor has adequate flexural strength and that a deflection calculation is unnecessary. This is similar to the procedure adopted in BS 8110[9], in that no deflection calculation for the fire condition is required. This is the approach that is adopted in BS 5950: Part 8.

Methods of predicting the deflections in fire conditions exist but they are complex and outside the scope of this publication.

5.2 Reinforcement

The arrangement of reinforcement within the concrete requires careful consideration both from the structural and economic standpoints. In many instances a standard reinforcing mesh, either A142 (6 mm diameter wires at 200 mm centres) or A193 (7 mm diameter wires at 200 mm centres) can be used, positioned towards the top of the slab. This will require support at close centres during construction. This is the most common form of reinforcement and its use is described in Section 5.4. The fire engineering design method permits the use of any arrangement of reinforcement provided it satisfies the normal design rules. The floor may be designed as simply supported with reinforcement being placed only to resist sagging or a combination of top and bottom reinforcement can be used. It is important that the mesh and bar reinforcement achieves the minimum ductility requirements of BS 4449: 1988[10], corresponding to a 12% minimum elongation at failure. This is because of the need to provide for sufficient rotation at the internal supports when developing the plastic failure mechanism of continuous slabs in fire conditions.

If this quality of reinforcement cannot be obtained then the designer should not place over-reliance on the hogging (negative) moment reinforcement. In such cases it is recommended that for more than 90 minutes of fire resistance the moment capacity of the hogging (negative) reinforcement is taken as not greater than that of the sagging (positive) moment reinforcement.

Some arrangements of reinforcement are illustrated in Figure 6.

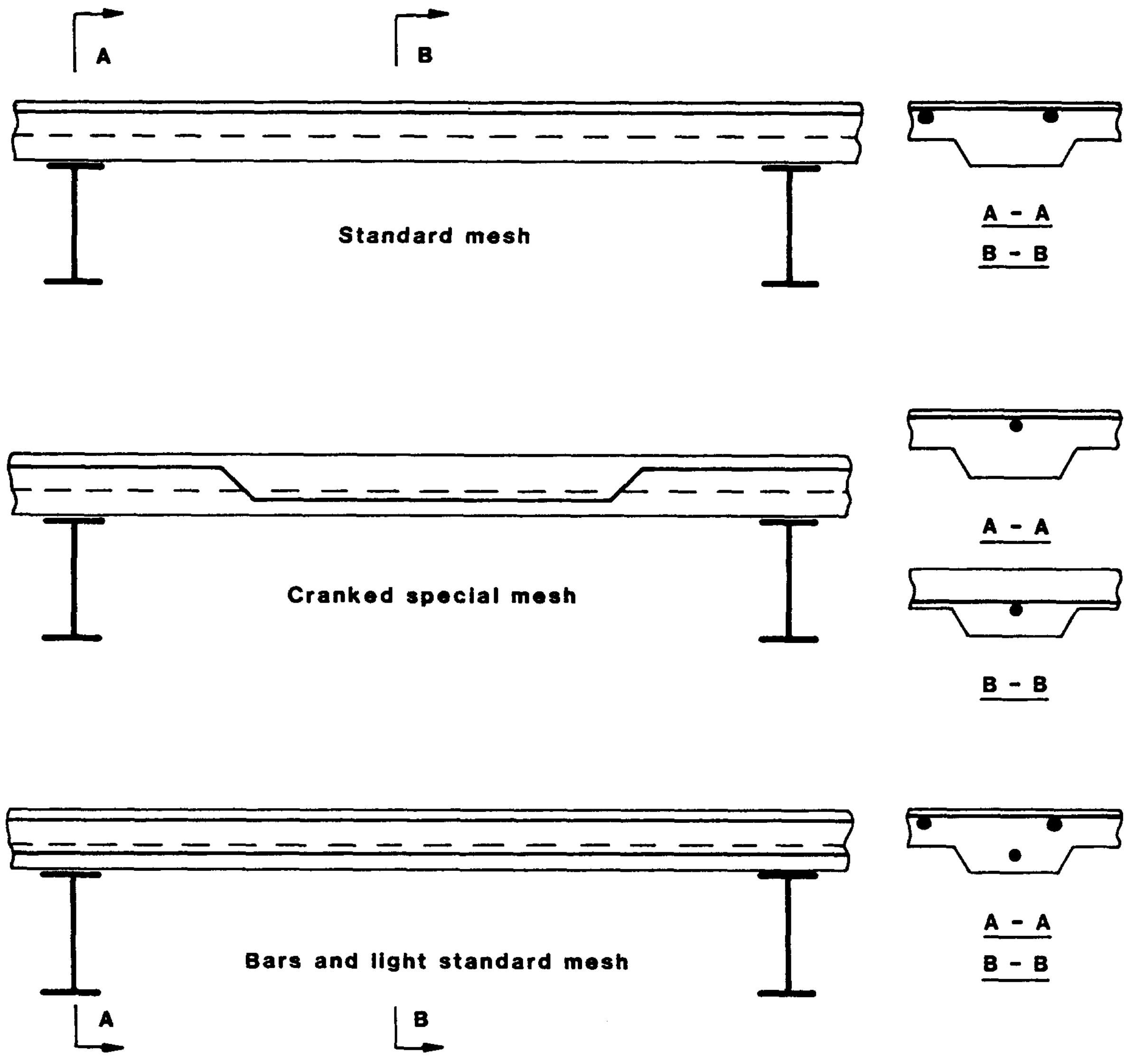


Figure 6 *Arrangement of reinforcement*
(Although trapezoidal deck is illustrated a dovetail deck could be used)

5.2.1 Special Mesh

Standard welded mesh has a pitch of 100 mm or 200 mm. Profiled steel decks are supplied in a range of pitches, typically up to 300 mm. To make best use of the reinforcement the mesh pitch should match the deck pitch. This can be achieved by using special meshes which can be supplied at little extra cost.

5.2.2 Draped or Cranked Mesh

Continuity can be achieved in a continuous design either by using 2 layers of mesh, by draping or by providing a shallow crank in a single layer of mesh. Meshes comprising small diameter wires often sag under their own weight. As the mesh diameter increases it will become necessary to physically bend the reinforcement to form a crank.

5.3 Fire Engineering Method

Design for fire resistance is based upon ultimate limit state principles. The floor slab is considered to act in bending either as a simply supported or continuous element.

5.3.1 Partial Factors

In carrying out the design the following partial safety factors are recommended:

- *Materials*
  Steel γ_{mr} = 1.0
  Concrete γ_{mc} = 1.3
- *Loads*
  Dead load γ_{fd} = 1.0
  Imposed load γ_{fi} = 1.0

In some situations, such as in office buildings, it is reasonable to use a partial factor for imposed load of less than unity. BS 5950: Part 8: 1990[5] allows the use of a partial factor of 0.8 for non-permanent imposed loads. The main reason for using a factor less than unity is that in most buildings the design imposed load is rarely achieved. Factors of less than unity are adopted in many countries where fire engineering methods are used. In the design examples, factors of unity are used for simplicity.

5.3.2 Material Strengths

The strengths of reinforcement and concrete (both normal and lightweight) may be obtained by multiplying the "room temperature" value by the factor, K_r, shown in Table 4.

For design at elevated temperatures the following stresses may be used.

Reinforcement:

$$\text{Design strength, } p_r = \frac{f_y K_r}{\gamma_{mr}} \qquad (1)$$

Concrete:

$$\text{Design strength, } p_c = \frac{0.67}{\gamma_{mc}} f_{cu} K_r \qquad (2)$$

where:

f_y = reinforcement yield strength

f_{cu} = characteristic concrete cube strength

K_r = factor from Table 4

0.67 = effective average stress factor for concrete (see Reference 13).

Table 4 *K_r material strength reduction factor*

Temperature (°C)	K_r material strength reduction factor		
	Reinforcement	Normal weight concrete	Lightweight concrete
<300	No reduction		
300	1.00	1.00	1.00
350	0.91	1.00	1.00
400	0.81	0.91	1.00
450	0.72	0.82	1.00
500	0.62	0.73	1.00
550	0.53	0.64	0.90
600	0.43	0.55	0.80
650	0.34	0.46	0.70
700	0.24	0.37	0.60

Data taken from Reference 13.

In addition, reinforcement and concrete should conform to the requirements of BS 8110: Part 1: 1985.

5.3.3 Concrete Depth

The minimum depth of concrete needed to satisfy the insulation requirements of BS 476 shall not be less than that shown in Tables 2 or 3 as appropriate. Alternatively it may be determined from a fire test on a similar construction. These depths have been revised from those given in earlier recommendations[1] following a review of recent test information.

5.3.4 Distribution of Temperature in a Floor Slab

The temperature of the reinforcement or concrete during a fire test may be determined from Table 5 (which should be read in conjunction with Figure 7). The information in this Table is taken from Reference 12 and is based upon solid slabs. Analysis of temperatures recorded in fire tests has shown this to be reasonable for design purposes, albeit slightly conservative.

Table 5 *Temperature distribution through a concrete slab*

Depth into slab (mm)	Temperature (°C) for fire resistance (hours) of:											
	½		1		1½		2		3		4	
	NW	LW	NW	LW	NW	LW	NW	LW	NW	LW	NW	LW
10	470	460	650	620	790	720	•	770	•	*	•	•
20	340	330	530	480	650	580	720	640	*	740	*	•
30	250	260	420	380	540	460	610	530	700	630	770	700
40	180	200	330	290	430	360	510	430	600	520	670	600
50	140	160	250	220	370	280	440	340	520	430	600	510
60	110	130	200	170	310	230	370	280	460	380	540	440
70	90	80	170	130	260	170	320	220	410	320	480	380
80	80	60	140	80	220	130	270	180	360	270	430	320
90	70	40	120	70	180	100	240	150	320	230	380	280
100	60	40	100	60	160	80	210	140	280	190	360	270

Data taken from Reference 13
NW *Normal weight concrete*
LW *Lightweight concrete*
• *indicates a temperature greater than 800 °C*
For any deck profile the depth into the concrete is measured normal to the surface of the steel deck.

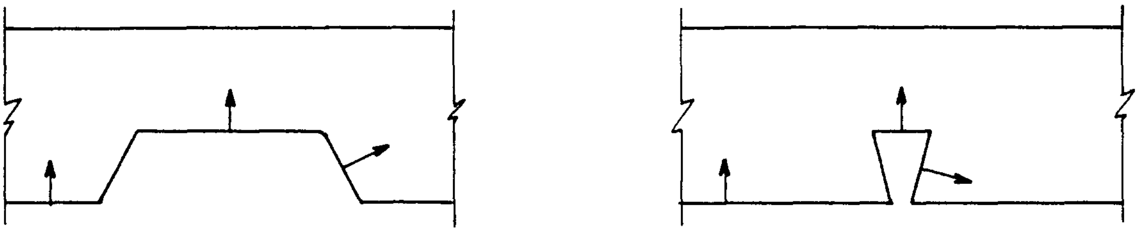


Figure 7 *Measurement of depth into the concrete*

5.3.5 Design Bending Moments for Continuous Construction

Design of continuous composite floors is based on a plastic failure mechanism and redistribution of moments may be assumed to take place in fire. However for fire resistance times greater than 90 minutes the hogging (negative moment) capacity should not be assumed to be greater than the sagging (positive moment) capacity.

The bending moment diagram for an internal span in fire conditions is as shown in Figure 8, and the condition for adequate plastic moment capacity is given by:

$$M_H + M_S \geq M_o \qquad (3)$$

where: M_H = Hogging moment capacity in fire per unit width
M_S = Sagging moment of capacity in fire per unit width
M_o = Free bending moment per unit width

$$= \frac{L^2}{8}\left(\gamma_{fd} w_d + \gamma_{fi} w_i\right) \qquad (4)$$

L = Span
w_d = Total dead load intensity
w_i = Imposed load intensity

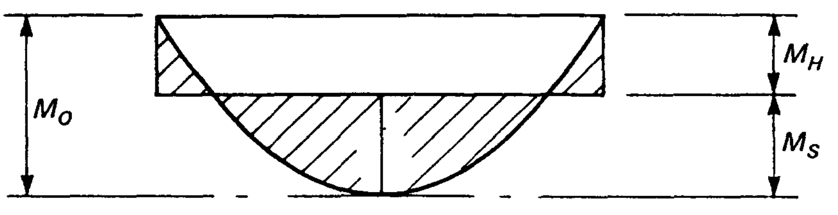


Figure 8 *Bending moment for internal span in a fire*

The bending moment diagram for an end span in fire conditions is as shown in Figure 9 and the condition for adequate plastic moment capacity is given by:

$$M_S + \frac{M_H}{2}\left[1 - \frac{M_H}{8M_o}\right] \geq M_o \qquad (5)$$

This is a more complex equation than for internal spans but as M_S, M_H and M_o are known the check can easily be carried out.

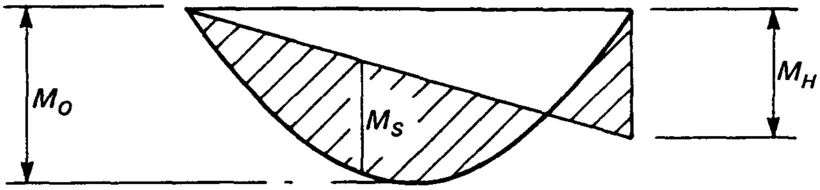


Figure 9 *Bending moment for an end span in a fire*

5.3.6 Design Examples

A design example illustrating the fire engineering method is given in the Appendix. The example illustrates the use of cranked mesh.

5.4 Simplified Method

This method consists of placing a single layer of standard mesh in the concrete. It was developed by CIRIA (see References 11 and 12). It differs from the fire engineering method in that calculations are not usually required. Since publication of the first edition this method has been extended up to 2 hours fire resistance based on the results of a large number of fire tests.

5.4.1 Loading

The imposed loads on the floor (live loads and finishes, etc.) should not exceed 6.7 kN/m². This maximum load may be increased in some circumstances (see Section 5.4.5).

5.4.2 Reinforcement

A142, A193 or A252 reinforcement satisfying the ductility requirements of BS 4449: 1988[10] (see Section 5.2) is required, the size of mesh depending on span and fire resistance time. The reinforcement should have top cover of between 15 mm and 45 mm. This means that it must be supported over the entire area. Reinforcement designed using the fire engineering method may in many areas rest directly on the deck.

5.4.3 Spans and Supports

Spans of up to 3.6 m may be used although this may be increased in some circumstances (see Section 5.4.5). The floors and reinforcement must be continuous over at least one internal support.

5.4.4 Design Tables

For trapezoidal decks the design data is given in Table 6. The data applies to deck profiles of 45 to 60 mm depth (see Figure 10). For deck profiles of depth D less than 55 mm and spans not greater than 3 m slab depths may be reduced by (55 - D) up to a maximum reduction of 10 mm. For deck profiles greater than 60 mm slab depths should be increased by (D - 60). Raised re-entrant details that protrude above the nominal top of the deck profile can normally be ignored provided they are not greater than 10 mm in height (Figure 10).

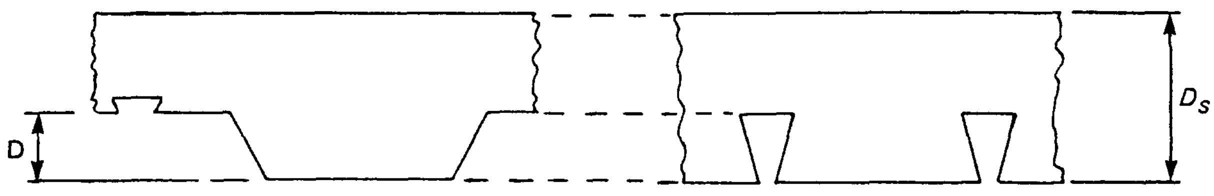


Figure 10 *Overall slab depth and deck depth*

Table 6 *Simplified design for trapezoidal decks*

Maximum span (m)	Fire resistance (hours)	Minimum dimensions			Mesh size
		t (mm)	D_S (mm)		
			NW	LW	
2.7	1	0.8	130	120	A142
3.0	1	0.9	130	120	A142
3.0	1½	0.9	140	130	A142
3.0	2	0.9	155	140	A193
3.6	1	1.0	130	120	A193
3.6	1½	1.2	140	130	A193
3.6	2	1.2	155	140	A252

NW *Normal weight concrete*
LW *Lightweight concrete*

Table 7 *Simplified design for dovetail decks*

Maximum span (m)	Fire resistance (hours)	Minimum dimensions			Mesh size
		t (mm)	D_S (mm)		
			NW	LW	
2.5	1	0.8	100	100	A142
2.5	1½	0.8	110	105	A142
3.0	1	0.9	120	110	A142
3.0	1½	0.9	130	120	A142
3.0	2	0.9	140	130	A193
3.6	1	1.0	125	120	A193
3.6	1½	1.2	135	125	A193
3.6	2	1.2	145	130	A252

NW *Normal weight concrete*
LW *Lightweight concrete*

For dovetail decks the design data is given in Table 7. The data applies to deck profiles of 38 to 50 mm depth. For deck profiles greater than 50 mm the slab depth should be increased by (D - 50).

In some circumstances the benefit of using greater slab depths can be taken into account (see Section 5.4.5).

In the design tables a minimum deck thickness *(t)* is given. This thickness is not critical, as in fire the deck heats up very quickly and retains only a small proportion of its strength. It should be considered as a practical limit.

5.4.5 Minor Variations

In the CIRIA publication a method is given which allows the spans given in Tables 6 and 7 to be varied by up to 0.5 m provided that the slab depth is not reduced and the bending capacity of the slab is not exceeded. The SCI have, using fire engineering techniques, devised a method of allowing the benefit of small increases in slab depth to be taken into account. It is not possible to reduce slab depths because the thermal performance of the floor would be adversely effected.

In considering variations, the starting point is the proven moment capacity which can be characterised by the free bending moment under test loading. This is given by:

$$M_o = (6.7 + w_s) \frac{L_o^2}{8} \tag{6}$$

where:

L_o = span, m (from Tables 6 or 7)
w_s = self weight, kN/m²
6.7 = total imposed load, kN/m²

Changes in imposed load, span and slab depth can then be made provided that:

$$M_o \times MDF \geq (w_i + w_s) \frac{L^2}{8} \tag{7}$$

where

MDF = moment depth factor from Table 8
w_i = revised total imposed load
w_s = revised self weight
L = revised span

The moment depth factor, *MDF*, is a measure of how much the moment capacity is increased for a given increase in overall slab depth. The total imposed load, w_i, should not exceed 12 kN/m² and the span may not be increased by more than 0.5 metres. However, the span may be reduced by any amount depending on the limit on imposed load. This has been introduced to ensure that shear failure does not occur in fire. In extending the earlier recommendations, conservative assumptions have been made in order to maintain the original levels of safety.

The second design example in the Appendix illustrates the use of the simplified method and how to apply variations covered by Equations 6 and 7.

Table 8 *Moment depth factor, MDF*

D_S (mm)	Moment depth factor, *MDF*, for an increase in D_S (mm) of:		
	10	20	30
100	1.08	1.17	1.25
110	1.08	1.15	1.23
120	1.07	1.14	1.21
130	1.07	1.13	1.20
140	1.06	1.13	1.19

5.4.6 Manufacturers' Design Tables

A number of steel deck manufacturers now publish design tables for a range of spans, loadings and slab depths. Design information prepared by the Steel Construction Institute for manufacturers may vary slightly from that derived using the information given in Table 6 or Table 7 when modified using the methods described above. This is due to the use of a more accurate assessment of the moment depth factor than that given in Table 8.

5.5 Comparison of Design Methods

The simplified method will almost invariably lead to the use of less reinforcement than the fire engineering method. This is because it is based directly on fire test results rather than on a theoretical structural model. In fire tests, materials are normally stronger than assumed in calculation and temperatures are generally lower than "design" temperatures. Also, although difficult to quantify, there is a strength contribution from the steel deck which is present in the tests but not included in calculations.

By way of compensation the fire engineering method allows greater flexibility in reinforcement layout, loading and range of fire resistance times. It also permits the use of thinner slabs, albeit with more reinforcement. For example, for one hour of fire resistance with a 50 mm deep trapezoidal deck and lightweight concrete, Table 2 gives 110 mm as the minimum slab depth required (50 mm deck plus 60 mm insulation thickness) whereas Table 6 gives a slab depth of 120 mm.

6. BEAMS SUPPORTING COMPOSITE FLOORS

Composite or non-composite beams will almost invariably require some form of applied fire protection to achieve the required fire resistance. The amount of fire protection would normally be specified using "Fire protection for structural steel in buildings"[15].

In 1990 SCI carried out a number of fire tests on composite beams. The test programme was sponsored by organisations representing a wide range of interests. An SCI Technical Report[14] on the research is available.

As a result of the research, recommendations for the fire protection of composite and non-composite beams were made. These include recommendations for the non-filling of the voids formed between the underside of the steel deck and the top flange of the beam. It was found that although additional heat entered the beam via the voids, the effect on moment capacity of the beam for periods of fire resistance up to 60 minutes is very small. Additionally, the inherent conservatism in the assessment of most fire protection materials is sufficient to allow for slight additional heating of the section.

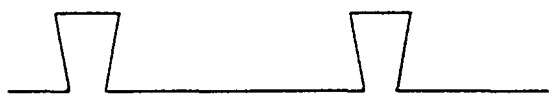
The main recommendations are summarised in Table 9.

Table 9 *Summary of recommendations*

TRAPEZOIDAL DECK

Construction	Fire protection on beam	Fire resistance (minutes)		
		Up to 60	90	Over 90
Composite Beams	BOARD or SPRAY (Assessed at 550°C)	No increase in thickness*	Increase thickness by 10% (or use thickness* appropriate to beam Hp/A + 15% whichever is less)	Fill voids
	INTUMESCENT (Assessed at 620°C)	Increase thickness* by 20% (or use thickness* appropriate to beam Hp/A + 30% whichever is less)	Increase thickness* by 30% (or use thickness* appropriate to beam Hp/A + 50% whichever is less)	Fill voids
Non-Composite Beams	All types	Fill Voids		

DOVETAIL DECK



Construction	Fire Protection on beam	Fire Resistance
Composite or Non-composite Beams	All types	Voids may be left unfilled for all fire resistance periods

* *Thickness is the board, spray or intumescent thickness given for 30, 60 or 90 minutes rating in "Fire Protection for Structural Steel in Buildings" (see Reference 15)*

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Use of Cranked Mesh.

A cranked special mesh with wires at 300mm. centres is used to match the deck profile. This is illustrated in figures 1.1 and 1.2.

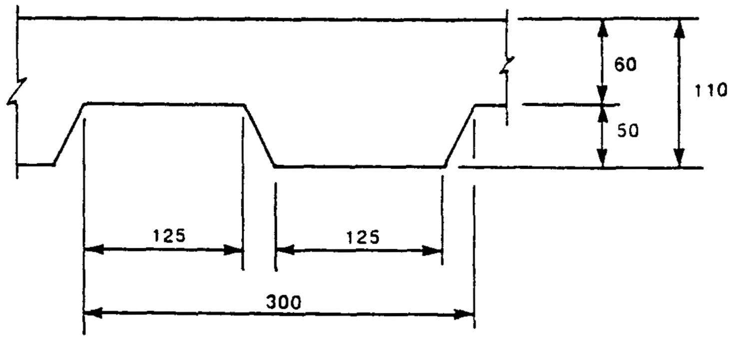

Figure 1.1 Deck profile and slab

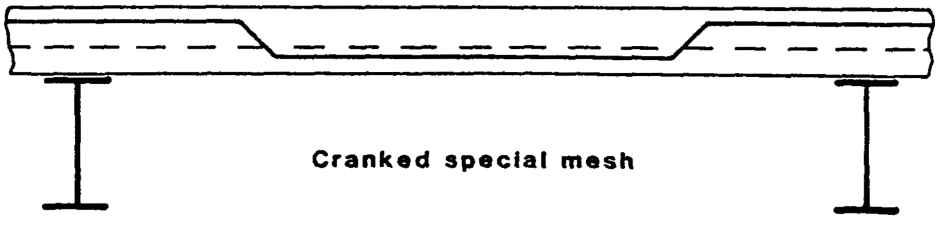

Figure 1.2 Schematic of cranked mesh

Assumed Design Parameters.

Span	$L = 3.0$ m.
Slab depth	$D_s = 110$ mm.
Concrete type	Lightweight.
Concrete strength	$f_{cu} = 30$ N/mm^2.
Deck type	Trapezoidal.
Deck depth	$D = 50$ mm.
Fire Resistance	$R = 1.0$ Hour.

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Assumed Loading.

Dead Loads: — kN/m²

	kN/m²
Selfweight of slab, w_s	1.75
Ceiling and Services	0.25
Total Dead Load, w_d	2.00 kN/m²

Imposed Loads:

Specified	3.00
Partitions	1.00
Total Imposed Load, w_i	4.00 kN/m²

Insulation

Concrete thickness above deck	60 mm.
Concrete type	LW.
Fire Resistance	1.0 Hour
Limit from Table 2 (Concrete thickness above deck)	60 mm.
	∴ OK.

Reinforcement.

Special cranked mesh with 7mm. diameter high yield wires at 300mm. centres to match the deck profile. Transverse wires or mesh must be provided to comply with the 0.1% minimum reinforcement requirement of BS. 5950: Part 4.

Load carrying capacity

The sagging and hogging moment capacities must be calculated and compared with the free bending moment.

Free bending moment, M_o

$$M_o = \frac{L^2}{8}\left(\gamma_{fd} \cdot w_d + \gamma_{fi} \cdot w_i\right) \text{ per unit width.} \quad \text{......Eq}^n.\ 4$$

$$= \frac{3^2}{8}\left(1.0 \times 2.0 + 1.0 \times 4.0\right)$$

$$= 6.75 \text{ kN.m. per metre width.}$$

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Sagging moment capacity, M_s.

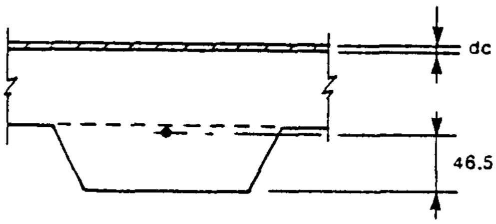

Figure 1.3 Section resisting sagging

The depth of reinforcement into the concrete is 46.5mm. For 1 hour fire resistance and LW. concrete Table 5 gives a temperature of 245°C. As this is below 300°C Table 4 gives no strength reduction ie. $K_r = 1.0$

Reinforcement design strength, p_r.

$$p_r = \frac{k_r . f_y}{\gamma_{mr}} \quad \text{.......Eq}^n. \ 1$$

$f_y = 460 \ N/mm^2$ (BS.8110 : Part 1).

$k_r = 1.0 \qquad \gamma_{mr} = 1.0$

$\therefore \ p_r = 460 \ N/mm^2$

Resistance of one wire, F_r

$$F_r = 460 \times \pi \times \frac{7^2}{4}$$

$$= 17703 \ N.$$

The centroid of the area of concrete at the top of the slab to balance this force must be found. Assume that the concrete is at full strength ie. $K_r = 1.0$

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Concrete design strength, p_c

$$p_c = \frac{0.67}{\gamma_{mc}} \cdot f_{cu} \cdot k_f \qquad \ldots\ldots \text{ Eq}^n. 2$$

$f_{cu} = 30 \text{ N/mm}^2$

$\gamma_{mc} = 1.3 \qquad k_f = 1.0$

$\therefore p_c = 15.46 \text{ N/mm}^2$

Depth of concrete, d_c

$$d_c = \frac{17703}{15.46 \times 300}$$

$$= 3.8 \text{mm.}$$

It can be seen from Tables 4 and 5 that this area of concrete is at a sufficiently low temperature for full strength to be assumed.

Internal lever arm, h

$$h = 110 - \frac{3.8}{2} - 46.5$$

$$= 61.6 \text{mm.}$$

Hence, $M_s = 61.6 \times 17703 \times 10^{-6}$ kN.m per 300mm.

$= 61.6 \times 17703 \times 10^{-6} \times \frac{1000}{300}$ kN.m. per metre.

$= 3.64$ kN.m. per metre width.

Hogging moment capacity, M_H

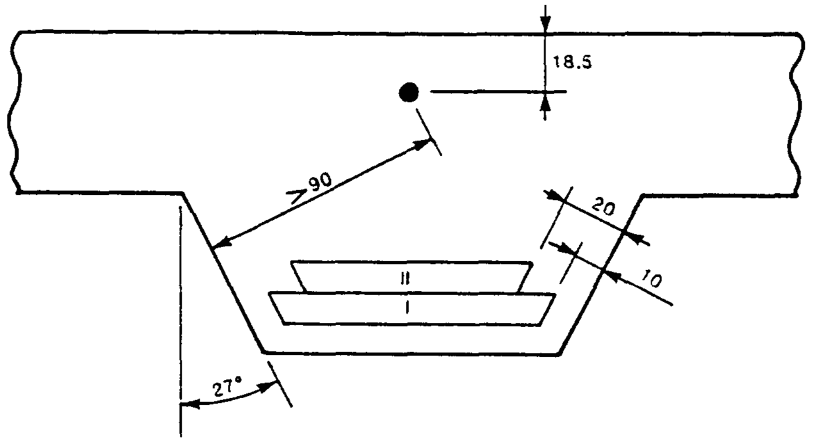

Figure 1.4 Section resisting hogging.

Assuming the top cover to the reinforcement is 15mm, the depth of the reinforcement into the concrete from the lower, fire exposed, face is in excess of 90mm. so full strength can be assumed (Tables 4 and 5).

Reinforcement design strength, p_r

p_r = 460 N/mm². (as in sagging)

Resistance of one bar, F_r

F_r = 17703 N.

The zone of concrete required to resist this force will not all be at the same temperature. The suggested procedure is to consider bands of concrete 10mm. thick. These are considered one by one until sufficient compressive resistance is obtained. In figure 1.4, 2 bands are shown. The outer 10mm. of concrete is normally ignored because it contributes little compressive strength.

Band I : 10 to 20 mm.
Average depth = 15 mm.

From Table 5 temperature = 550°c.

From Table 4 k_r = 0.9

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Concrete design strength, p_c

$$p_c = \frac{0.67}{\gamma_{mc}} \cdot f_{cu} \cdot k_f \quad \ldots\ldots \text{ Eq}^n.\ 2$$

$f_{cu} = 30\ N/mm^2$

$\gamma_{mc} = 1.3 \qquad k_f = 0.9$

$\therefore\ p_c = 13.91\ N/mm^2$

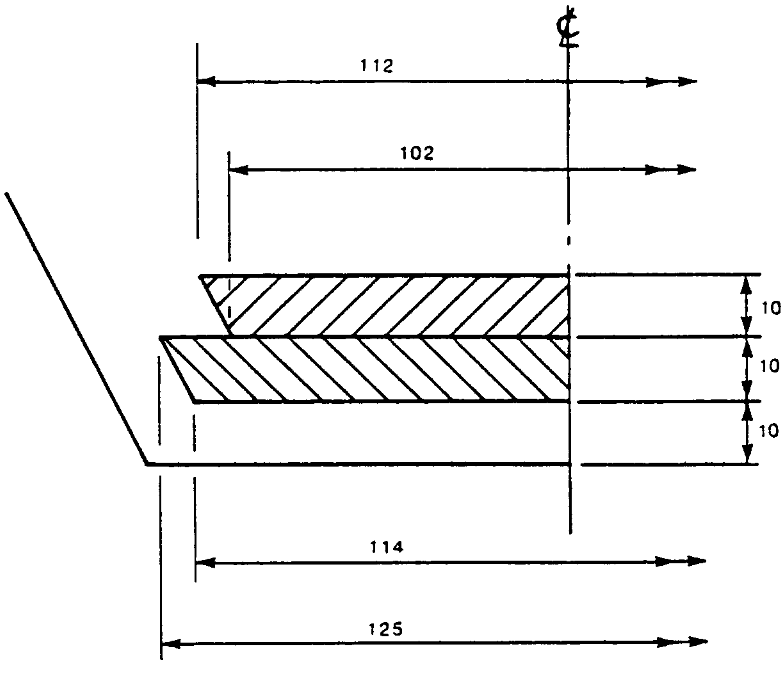

Figure 1.5 Determination of concrete area.

Area of concrete in band I (Figure 1.5).

$= \frac{10}{2}(125 + 114)$

$= 1195 mm^2$

Resistance of band I $= 1195 \times 13.91$ N.

$= 16622$ N.

This is less than the resistance of the reinforcement so band II must be considered.

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Band II : 20 to 30mm.
Average depth = 25mm.

From Table 5 temperature = 430°C.

From Table 4 k_p = 1.0

∴ p_c = 15.46 N/mm²

Area of concrete in band II = $\frac{10}{2}(112 + 102)$

= 1070 mm²

Resistance of band II = 1070 × 15.46
= 16542 N.

The combined resistance of band I and band II is greater than the resistance of the reinforcement so only a portion of band II is required.

Depth of band II required

$$= 10 \times \frac{(17703 - 16622)}{16542}$$

= 0.65 mm.

The centroid of the concrete required in compression is therefore approximately

$$10 + (10 + 0.65) \times \frac{1}{2}$$

= 15.33 mm. from the soffit of the deck

Internal lever arm, h

$h = 110 - 15.33 - 18.5$

= 76.2mm.

$\therefore M_u = 76.2 \times 17703 \times 10^{-6}$ kN.m per 300 mm.

$= 76.2 \times 17703 \times 10^{-6} \times \frac{1000}{300}$ kN.m per metre.

= 4.50 kN.m per metre width.

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__Capacity check:__

Internal spans

$$M_S + M_H \geqslant M_o \qquad \ldots\ldots \text{ Eq}^n.\ 3$$

$$3.64 + 4.50 = 8.14$$

$$M_o = 6.75$$

∴ ok.

End spans

$$M_S + \frac{M_H}{2}\left(1 - \frac{M_H}{8M_o}\right) \geqslant M_o \ldots \text{Eq}^n.\ 5$$

$$3.64 + \frac{4.5}{2}\left(1 - \frac{4.5}{8 \times 6.75}\right) = 5.70$$

$$M_o = 6.75$$

∴ Unsatisfactory.

The end spans therefore require additional reinforcement. This is not an uncommon result. The capacity may be increased by providing an additional wire of 5mm. diameter alongside the longitudinal mesh bars. The sagging moment capacity is then increased to 5.5 kN.m. and the end span is satisfactory.

__Extent of reinforcement.__

The position at which the mesh, continuing past the supports may be curtailed must be checked. The requirements of BS. 8110 should generally be followed

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The Simplified Method

A "dovetail" deck is required to carry a load greater than 6.7 kN/m² but with a slab depth greater than the minimum and with a span less than the maximum specified in Table 7.

Assumed design parameters:

Span	L = 3.0 m.
Slab depth	D_s = 145 mm.
Concrete type	Normal Weight
Concrete strength	f_{cu} = 30 N/mm².
Deck type	Dovetail
Deck depth	D = 50 mm.
Fire resistance	R = 1.5 hours.

Assumed Loading	kN/m²
Selfweight of slab	w_s = 3.27
Ceiling and services	= 0.25
Specified imposed load	= 10.00
Partitions	= 1.00

The total imposed load is therefore 11.25 kN/m². which is greater than 6.7 kN/m² on which the design Tables (6 and 7) are based.

For a 3.0 metre span Table 7 gives

D_s = 130 mm.
and A142 reinforcement is required.

The self weight of this 130mm. slab is 2.92 kN/m².
Consider the effect of increased slab depth.
New slab depth = 145mm.
New slab weight = 3.27 kN/m².

From Table 8 an increase of slab depth from 120mm. to 145 mm. gives an increase in moment capacity given by

MDF = 1.10

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Original free bending moment

$$M_o = (6.7 + 2.92)\,\frac{3.0^2}{8} \quad \ldots\ldots \; Eq^{n.}\ 6$$

$= 10.82$ kN.m. per metre width.

Revised moment capacity.

$= 1.10 \times 10.82$
$= 11.9$ kN.m. per metre width.

Applied moment
$= (w_i + 3.27) \times \frac{3^2}{8}$

Equating moments gives

$w_i = 7.31$ kN/m^2.

This is less than the required 11.25 kN/m^2. so try increasing the reinforcement to A193.

For a 3.6 m. span Table 7 gives

$D_s = 135$ mm.
and A193 reinforcement.

The selfweight of this 135mm. slab is 3.04 kN/m^2.
From Table 8 an increase of slab depth from 135mm. to 145mm. gives an increase in moment capacity given by

MDF = 1.065

Original free bending moment

$$M_o = (6.7 + 3.04) \times \frac{3.6^2}{8}$$

$= 15.78$ kN.m per metre width.

Revised moment capacity
$= 1.065 \times 15.78$
$= 16.81$ kN.m.
Equating moments gives
$w_i = 11.66$ kN/m^2.

> 11.25 kN/m^2.

Therefore A193 mesh is satisfactory